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PAPERS

BASE COURSE DRAINAGE FOR AIRPORT PAVEMENTS

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Synopsis

Base courses beneath airport pavements may become saturated under some conditions. The principal causes for base saturation were determined from observations on a number of airfields. An approximate theoretical analysis for the drainage of a saturated base course was developed and compared with the results of laboratory model tests and full scale field tests. These comparisons show that the formulas which are proposed are satisfactory for the design of base drainage of wide pavements.

Introduction

It is generally recognized that saturated base courses are detrimental to the life of pavements because they are not as strong as drained bases. Water forced upward through the pavements at joints and cracks by traffic is known to produce early disintegration of bituminous mixtures and pumping at joints of concrete pavements. Therefore, base-course drainage is often necessary to remove a major cause of failure in airport pavements. It is the purpose of this paper to analyze the conditions under which base courses become saturated and to develop and test a procedure for the design of base drainage.

Observations on Airport Pavements

To determine the conditions under which base courses may become saturated, observations were made from 1945 to 1947 by the Corps of Engineers, United States Department of the Army, at airfields in the continental United States. These observations showed that base courses of airfield pavements may become fully saturated under certain conditions. Most of the observations were

Note.—Written comments are invited for publication; the last discussion should be submitted by December, 1951.

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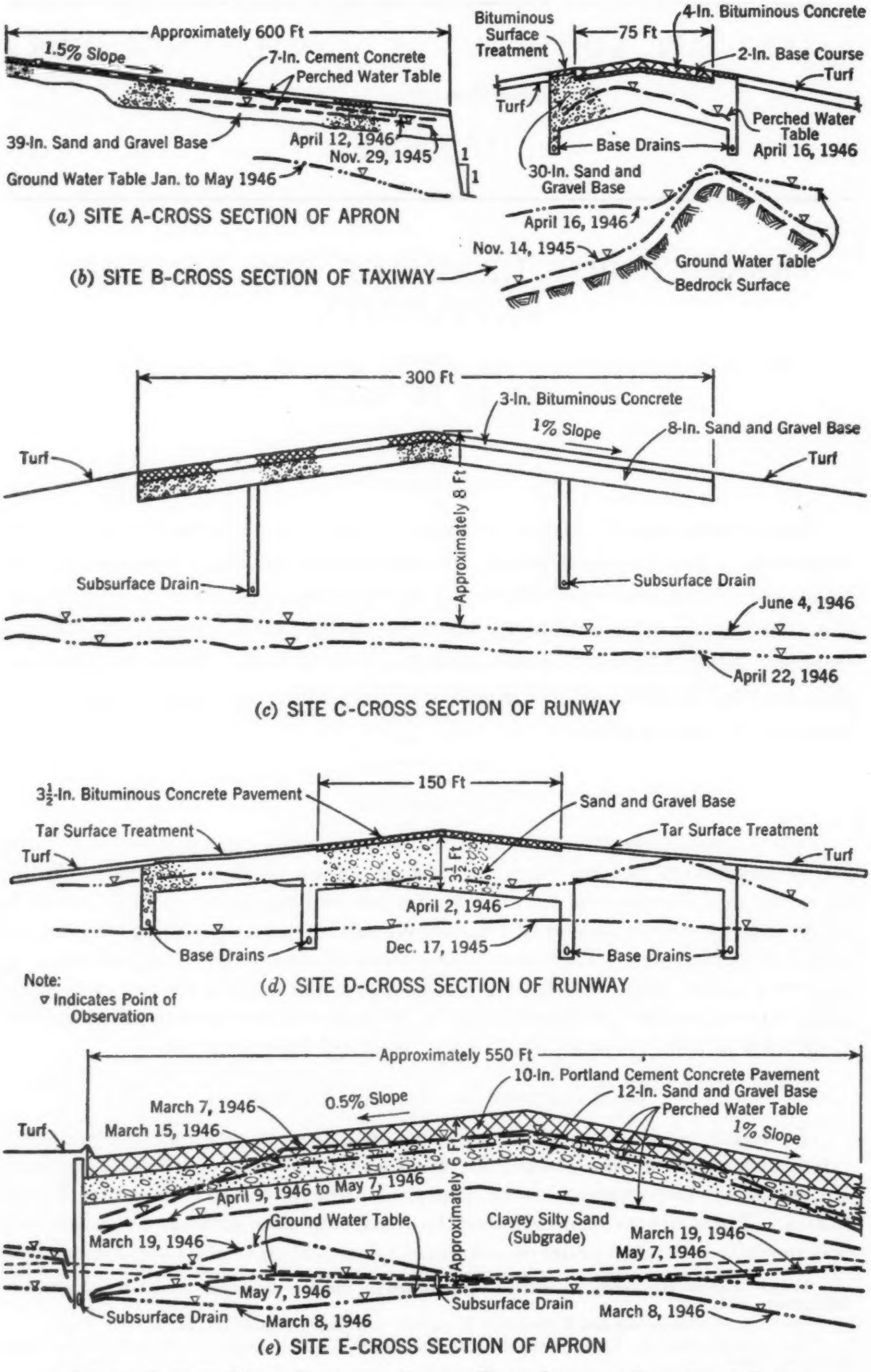


Fig. 1.—Perched Water Level and Ground-Water Level at Observation Sites

limited to the two principal causes for the saturation of base courses—frost action and infiltration through the pavement. At six airfields, in the states of Maine, Michigan, Wisconsin, North Dakota, and South Dakota, detailed observations were made of ground-water levels in the subgrade, and of perched water tables in the base courses beneath both concrete and bituminous pavements. The discharge of base-course drains was measured at several of these fields.

General Conditions at Site A.—In Fig. 1(*a*) the test area designated site A is the part of an airfield located in northern Maine. The base course consists of a well-graded sand and gravel, with material larger than about 3 in. removed, and containing 3% to 5% of material smaller than a No. 200 mesh sieve. The coefficient of permeability of the base course material averages 27×10^{-4} cm per sec. No base drains are provided. However, on the low edge of the pavement, the base course is carried through the shoulder, thus permitting drainage.

The subgrade beneath the base course is fill, about 5 ft deep, consisting of compact, silty sand, and gravel, with a coefficient of permeability averaging 1×10^{-6} cm per sec. A similar soil underlies this fill, and shale bedrock is found at a depth of more than 20 ft.

Test pits excavated during the winter of 1945–46 indicated frost penetration to a depth of about 6 ft from the pavement surface. Ice lenses were observed throughout the depth of frozen subgrade, but no ice segregation was observed in the base course. The frost action in the subgrade heaved the pavement an average of 0.2 ft.

The observation wells indicated that during the fall and winter the depth of the ground-water table in the subgrade fluctuated from 7 ft to 12 ft below pavement surface. During the fall, a perched water table was observed in the base course, as shown in Fig. 1(*a*). In the spring as soon as the base had thawed, the observation wells again indicated a perched water table in the base and a rising water table in the subgrade. The perched water table in the base rose quickly as thawing progressed, almost to the surface of the pavement; then, slowly it drained and by the end of June it had dropped to about the middle of the base course.

Discussion of Base Drainage at Site A.—In the fall of the year the average water content of the base above the perched water table was found to be about 6%, which may be assumed to be held by capillarity. At an average unit dry weight of 140 lb per cu ft, the base would be fully saturated at an average water content of 8%. Thus, only 2% by dry weight would be required to change this bank-run sand and gravel base from the drained to the fully saturated state. For the base thickness of 39 in., these 2% are equivalent to 10 gal of water per square yard.

Assuming that the average observed heave of the pavement surface during the winter (0.2 ft) is equal to the total thickness of ice lenses in the subgrade, the quantity of water liberated by thawing of the subgrade would be about 13 gal per sq yd. It may reasonably be assumed that a major portion of this quantity drained upward into the base. Thus, sufficient water was available to saturate the base during the spring.

During the fall the saturation of the base is believed to have resulted from infiltration of water through joints and cracks in the pavement. An estimate of

the quantity of infiltration may be made by assuming that the joint width is 0.1 in. and that atmospheric pressure is maintained at the under side of the pavement. Assuming laminar flow, the quantity that would flow through the joint is computed to be 0.15 cu ft per sec per ft of joint length. At this rate and with a joint spacing of 10 ft, the base would become saturated in about 1 min. A similar computation using a joint width of 0.01 in. indicates that the base would become saturated in about 20 hr.

The preceding discussion indicates that saturation of the base at this site is possible, during the spring, by water from melting ice lenses in the subgrade, and that saturation can occur at other times by infiltration through joints and cracks in the pavement.

Observations at Site B.—At site B (Fig. 1(*b*)), also located in northern Maine, observations were made of the ground-water table and perched water table in the base course beneath a taxiway with a bituminous concrete surface. The base course and subgrade were similar to site A, except that the height of filled subgrade was less. Subsurface drains were provided at the edge of the pavement. At this site the highest and lowest positions of the ground-water table in the subgrade, over a period of one year, were above and almost parallel to the surface of the underlying bedrock. During the spring, immediately after the base course thawed, a perched water table was observed in the base. This water table gradually dropped, and within three weeks after thawing was complete, the base course was drained.

Frost action in the subgrade resulted in a heave of the pavement surface of about 0.25 ft. Over most of its area the bituminous concrete pavement was free from cracks. Therefore, it is believed that the perched water table in the base resulted from upward seepage of water from the thawing subgrade.

Observations at Site C.—Site C (Fig. 1(*c*)), located in Massachusetts, was a part of a bituminous concrete paved runway with a sand and gravel base constructed on a clean sand subgrade. Before construction, the ground-water table was within about 1 ft of the ground surface. During construction the water table was lowered by means of open ditches. Those ditches, located along runway edges, were later replaced with closed joint pipe with surface inlets, whereas those located in areas between runways were maintained as open ditches. In addition, subsurface drains were installed beneath the pavement as shown in Fig. 1(*c*) to control the ground-water table.

Observations of the ground-water table and of the subsurface drains during the spring of 1946 indicated that the ground-water table never rose above the invert of these drains, as shown by highest and lowest positions of the ground-water profiles in Fig. 1(*c*). For this subgrade (which has a coefficient of permeability of about 10×10^{-4} cm per sec) the open ditches in the areas between runways are apparently adequate to maintain a reasonably constant ground-water table. The nearest open ditch to the location of the observations was at a distance of about 1,000 ft.

Observations at Site D.—Site D (Fig. 1(*d*)), in central Maine, was of a runway paved with bituminous concrete with a gravel shoulder surface-treated with tar. The base course beneath the pavement and in the treated shoulder consists of a well-graded, slightly silty sand and gravel. The subgrade is a gray-blue, silty

clay. At the location of the observations, the bottom of the base course is several feet below the original ground surface. Four lines of subsurface drains are installed.

Observations of the ground-water table in 1946 and 1947 indicated that during the summer, fall, and winter the water table became progressively lower, reaching the lowest position in December. During the spring, as the subgrade was thawing, a water table was observed in the base course, as shown for April 2 in Fig. 1(*d*). In the shoulder area the water table sometimes rose as high as the ground surface. When the subgrade was completely thawed, the ground-water table, for a short period, was at about the same elevation as the water table in the base during thawing. Then the ground-water table gradually dropped, and by August both the base course beneath the pavement and the surface-treated shoulder was substantially drained.

During the spring, the water in the base in the shoulder areas frequently stood at a higher elevation than in the base beneath the paved area, as shown by the ground-water surface for April 2. In addition a close correlation was observed between precipitation and discharge of subsurface drains in the shoulder. Therefore, it may be concluded that the high water table beneath the shoulder area was due largely to the infiltration of precipitation into the base through the bituminous surface treatment. On the other hand, it is believed that the water in the base beneath the paved area is principally water released by melting ice in the subgrade, augmented perhaps by some infiltration through cracks in the pavement.

Observations at Site E.—Site E (Fig 1(*e*)), in Michigan, consists of a part of a Portland cement concrete apron with a sand and gravel base course and a clayey-silty sand subgrade. Subsurface drains extend around the periphery and beneath the apron.

Observations of the water table in the subgrade indicated that it varied from slightly above the elevation of the subsurface drains to the elevation of the lowest invert. The water table was highest following the frost melting period. No ice segregation was observed in the subgrade; however, heaving of the pavement surface, averaging 3/4 in., was measured. A perched water table was observed in the base course during the thawing period, as shown for March 7 and March 15. Therefore, it is believed that most of the water in the base during the thawing period must have been caused by infiltration of surface water through joints and cracks in the pavement. Actually there was an almost continuous flow of water across the pavement during this period, resulting from the melting of a large snow bank piled near the crown of the apron.

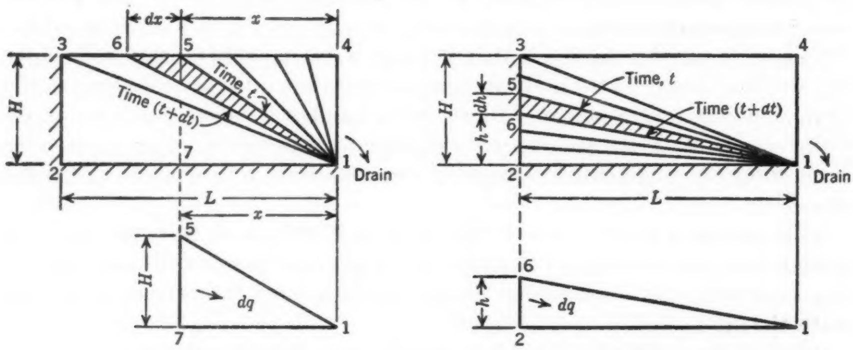
Conclusions from Field Observations.—Based on these and other observations it was concluded that during the thawing period ice segregation in a subgrade may be the cause of saturation of an overlying, free-draining base. It was also concluded from some of the preceding observations that infiltration of surface water through pavement cracks, joints, or through a pervious pavement, such as certain types of penetration macadam pavements, may cause saturation of a free-draining base overlying a relatively impervious subgrade. Other causes for the saturation of bases may be inundation of the pavement, such as might

occur at an airport located in an area that might become flooded, or where the natural water table may rise above the bottom of the base course.

Theoretical Analysis of Base Drainage

Drainage of a saturated base is essentially a problem in nonsteady flow (also called transient flow) with a free surface. Present development of mathematical tools does not permit a rigorous solution of such problems. Therefore, any attempt at a mathematical solution must be based on simplifying assumptions.

Analysis of Horizontal Base.—The assumptions on which the mathematical analysis is based are illustrated in Fig. 2. The center line of the base course and the bottom of the base course are considered replaced by impervious boundaries as shown by the cross-hatched sides of the rectangle. Open discharge is assumed at the right side. At the start of drainage (elapsed time of zero) the base course is assumed saturated, and the right side suddenly opened for free



(*a*) *U* EQUAL TO OR GREATER THAN 50%

(*b*) *U* EQUAL TO OR LESS THAN 50%

Fig. 2.—Assumed Progress of Free Water Surface—Horizontal Base

drainage. The free water surface is assumed to remain a straight line that changes with time, as illustrated.

For the purpose of analysis, the drainage process is divided into two parts: (1) the first part, Fig. 2(*a*) in which the free surface gradually changes from position 1–4 to position 1–3; and (2) the second part, Fig. 2(*b*) in which the free surface changes from position 1–3 to position 1–2. For the first part of the drainage process, the differential equation can be set up by considering the position of the free surface at elapsed time t and then at time $(t + dt)$. In the time element dt, the quantity dq discharged per unit width is equal to the area of the narrow, shaded triangle 1–5–6, multiplied by the effective porosity n_e. The effective porosity n_e is the ratio of the volume of voids that can be drained to the total volume of soil. It is assumed constant, that is, independent of the height above the impervious boundary.

$$dq = \frac{H\, n_e}{2}\, dx \qquad \text{(1)}$$

The flow through volume 1–5–7 is computed by means of Darcy's law. The simplest assumption that could be made is to use $H/2$ as the average area per unit of width through which flow takes place and to assume an average or effective hydraulic gradient of H/x. Then the rate of flow could be expressed by

$$\frac{dq}{dt} = k\,\frac{H}{2}\,\frac{H}{x} = k\,\frac{H^2}{2\,x} \qquad (2a)$$

In this form, the equation is identical with Dupuit's formula for steady seepage through a homogeneous earth dam.

A greater flexibility can be introduced in the solution of this problem by using an effective hydraulic gradient equal to $\dfrac{H}{c_1\,x}$ and an average area per unit of width equal to H/c_2, in which c_1 and c_2 are quantities that will be assumed constant during the entire drainage process. Substituting $c = c_1\,c_2$, one obtains

$$\frac{dq}{dt} = k\,\frac{H^2}{c\,x} \qquad (2b)$$

Whereas in Eq. 2*a* the constant c is equal to 2, a known—or rather assumed—quantity, it should be noted that in Eq. 2*b* the quantity c represents an unknown parameter that must be determined by other means—for instance, from model tests. Combining Eq. 1 and Eq. 2*b*, a simple differential equation is derived, the solution of which leads to

$$t = \frac{c\,n_e\,x^2}{4\,k\,H} \qquad (3)$$

The progress of drainage is best defined as the ratio of the drained area (triangle 1–4–5) to the total area (rectangle 1–2–3–4). This dimensionless ratio will be called the degree of drainage

$$U = \frac{\text{Drained Area}}{\text{Total Area}} \qquad (4)$$

or expressed as a percentage: $U\% = 100\,U$. In addition to U, it is expedient to introduce another dimensionless quantity

$$T = \frac{2\,t\,k\,H}{c\,n_e\,L^2} \qquad (5)$$

that will be called the time factor.

Introducing U and T in Eq. 3 gives the formula for the progress of drainage:

$$T = 2\,U^2 \qquad (6)$$

Eq. 6 is valid for $U \leqq 0.5$ (or $\leqq 50\%$). By a similar derivation one obtains the solution for the second half of the drainage process, for which the variable triangle 1–2–6 has a constant base length L and a variable height h (Fig. 2(*b*)).

The starting equations are:

$$dq = \frac{L\,n_e}{2}\,dh \quad \text{(7a)}$$

$$dq = k\,\frac{h}{2}\,\frac{h}{x}\,dt = k\,\frac{h^2}{2\,x}\,dt \quad \text{(7b)}$$

and

$$dq = k\,\frac{h^2}{c\,x}\,dt \quad \text{(7c)}$$

The final solution can be expressed by the following relation between the dimensionless quantities T and U

$$T = \frac{U}{2 - 2\,U} \quad \text{(8)}$$

Eq. 8 is valid for $U \geqq 0.5$ (or $\geqq 50\%$).

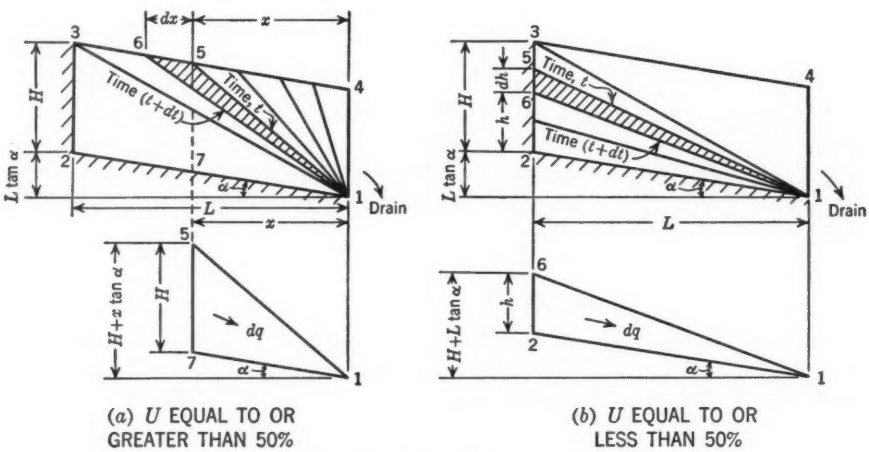


Fig. 3.—Assumed Progress of Free Water Surface—Sloping Base

Analysis of Sloping Base.—The assumptions on which the following analysis is based are similar to those made for the analysis of the horizontal base and are illustrated in Fig. 3. The mathematical derivation, however, is not quite as simple. For the first half of the drainage process (Fig. 3(a)) the starting equations are

$$dq = \frac{H\,n_e}{2}\,dx \quad \text{(9a)}$$

$$dq = k\,\frac{H}{2}\,\frac{H + x\tan\alpha}{x}\,dt = k\,\frac{H^2 + H\,x\tan\alpha}{2\,x}\,dt \quad \text{(9b)}$$

and

$$dq = k\,\frac{H^2 + H\,x\tan\alpha}{c\,x}\,dt \quad \text{(9c)}$$

To simplify the presentation of the solution, it is desirable to introduce (in addition to T and U as previously defined) a third dimensionless quantity that

will be called the slope factor.

$$S = \frac{H}{L \tan \alpha} \quad \text{.......................} (10)$$

in which α is the angle of slope of the base layer.

The final solution can be expressed by the following relationship among the dimensionless quantities T, U, and S:

$$T = 2\,U\,S - S^2 \ln \frac{S + 2\,U}{S} \quad \text{.................} (11)$$

which is valid for $U \leqq 0.5$ (or $U \leqq 50\%$). By a similar derivation (Fig. 3(*b*)) the solution for the second half of the drainage process is obtained. The starting equations and the final solution are as follows:

$$dq = -\frac{L\,n_e}{2}\,dh: \quad \text{.....................} (12a)$$

$$dq = k\,\frac{h}{2}\,\frac{h + L \tan \alpha}{L}\,dt = k\,\frac{h^2 + h\,L \tan \alpha}{2\,L}\,dt.: \quad \text{........} (12b)$$

or

$$dq = k\,\frac{h^2 + h\,L \tan \alpha}{c\,L}\,dt \quad \text{..................} (12c)$$

and

$$T = S + S \ln \frac{2\,S - 2\,U\,S + 1}{(2 - 2\,U)\,(S + 1)} - S^2 \ln \frac{S + 1}{S}: \quad \text{.........} (13)$$

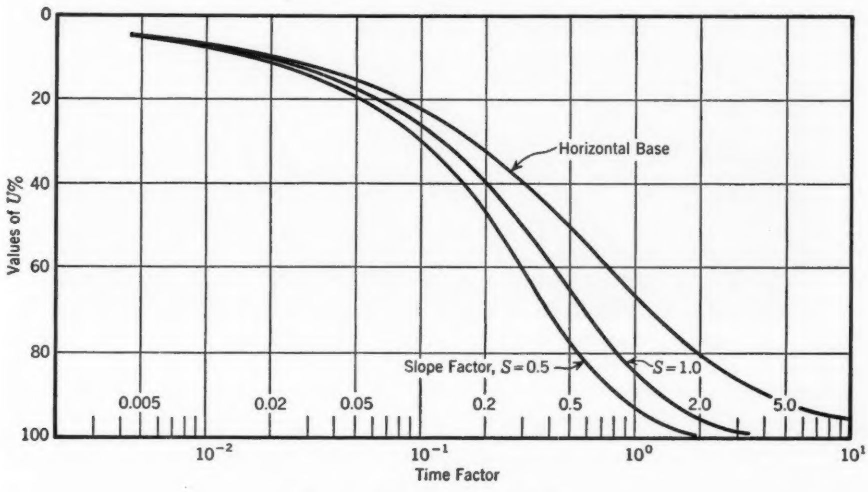


FIG. 4.—TYPICAL TIME CURVES

Discussion of Theoretical Results.—Eqs. 6 and 8 are shown graphically for the horizontal base in Fig. 4; and Eqs. 11 and 13 are shown for an inclined base having a slope factor of $S = 0.5$ and 1.0. For the most purposes it is convenient to plot such U versus T curves on semilogarithmic paper. Since the time factor

(T) is directly proportional to the time (t), the effect of a slope on the rate of drainage can be readily compared in Fig. 4. For example, a sloping base, with a slope factor of $S = 0.5$, requires about one half the time of a horizontal base to obtain 50% drainage, and about one quarter of the time to reach 90% drainage.

It should be noted that, for a horizontal base, one U versus T curve represents the solution for all possible combinations of k, n_e, H, L, and c, since all these variables, and also the time t, are contained in the time factor T. To use theoretical T versus U curve for an actual case, one need only transform the T-scale into a time scale using Eq. 5.

Laboratory Model Tests

In the past the flow of a viscous fluid between parallel plates has been used to study problems of steady seepage. At first the application of this principle to problems of nonsteady flow was found unsatisfactory because any viscous fluid

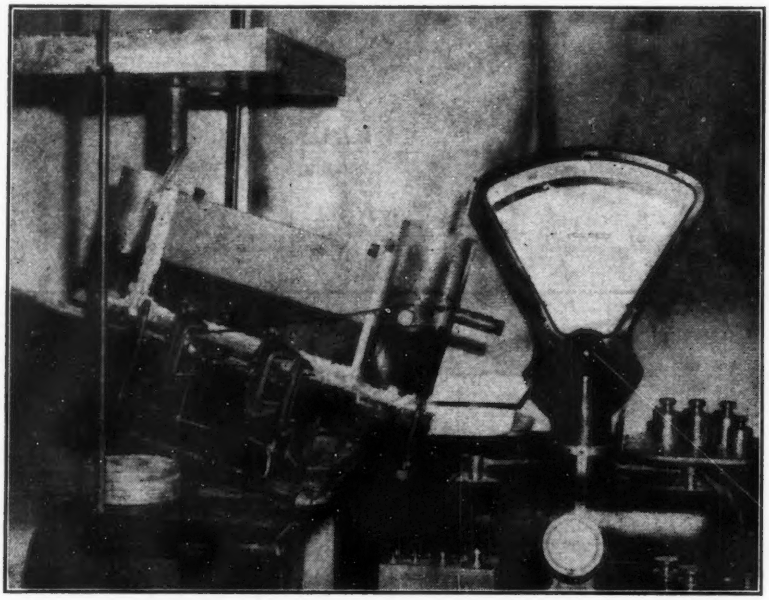


Fig. 5.—Constant Velocity Apparatus for Permeability Tests

will adhere to the glass plates. By extensive experimentation this obstacle was overcome in the following manner:

(a) The glass plates were covered with a very thin film of a soft grade of paraffin, by dipping the plates into melted paraffin and withdrawing them quickly after the plates had reached approximately the temperature of the paraffin. Then the paraffin was removed from the outside surface to increase visibility.

(b) Glycerine was used as viscous liquid, and about 10 gm of sodium hydroxide was added to each 100 cc of glycerine to increase the viscosity; a small amount of phenolphthalein was used to produce a light red color.

The models consisted essentially of two parallel glass plates with an inside space of ¼in., mounted on a tilting frame. The bottom and the left end of these models were closed, and drainage was permitted through the right end into a vessel that rested on an accurate indicating scale. Model dimensions and fluid viscosity were so controlled that flow was laminar at all times. By proper lighting of these models, the surface of the colored glycerine could be made clearly visible through the coated plates. The glycerine was prevented from discharging during filling by a temporary stop at the open end. After the model

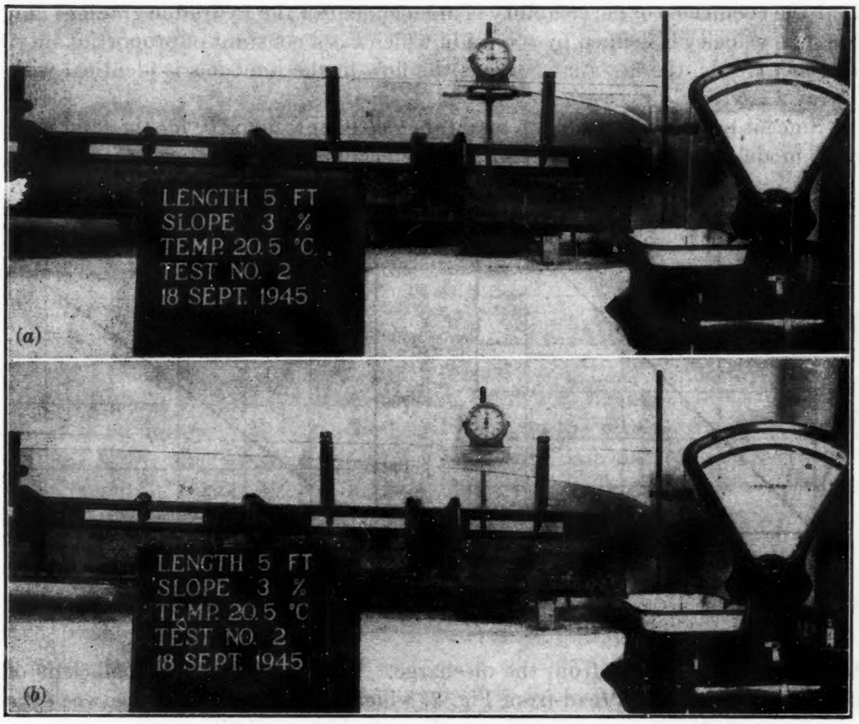


FIG. 6.—VISCOUS FLUID MODEL TEST

(a) 15 SECONDS AFTER START OF DRAINAGE

(b) 2 MINUTES AFTER START OF DRAINAGE

was filled, it was inclined quickly to the desired slope, and the temporary stop was removed.

During a test photographs were taken periodically, and measurements were made of the weight of the drained fluid, the position of the free surface, and the temperature of the fluid. In addition, samples of the drained fluid were taken and tested for viscosity. The coefficient of permeability that was effective in the model tests was determined by means of direct permeability tests using two different types of apparatus. Most of the tests were performed on the constant velocity type apparatus shown in Fig. 5, the plate treatment and spacing being

the same as that used in the models. Several tests were performed on a free discharge apparatus similar to that shown in Fig. 6 with the left side connected to a reservoir in which a constant head was maintained. For each permeability test the Saybolt-Fural viscosity was determined on a sample of the fluid taken from the discharge. The relationship between velocity and hydraulic gradient, as obtained from these tests and reduced to a constant viscosity, is plotted in Fig. 7. The values given are for a plate spacing of 0.635 cm and have been reduced to a constant Saybolt-Fural viscosity of 400 sec per 60 cc. The practically linear relationship between hydraulic gradient (i) and velocity (v) shows that the coefficient of permeability is independent of the hydraulic gradient and that the velocity is defined by $v = k\,i$ in which k is a constant of proportionality. In other words, the law that governs the flow in these models is identical with Darcy's law.

Since it proved impossible to maintain a constant viscosity for the fluid used in all model tests, the viscosity had to be determined simultaneous to each model

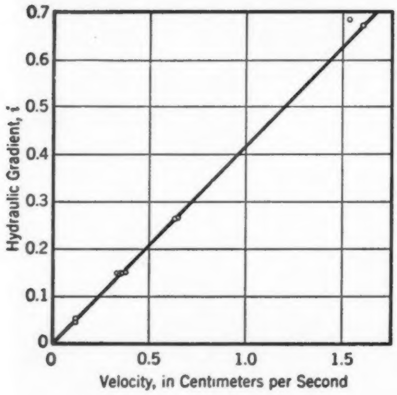


Fig. 7.—Test Results Obtained with Constant Velocity Permeability Apparatus

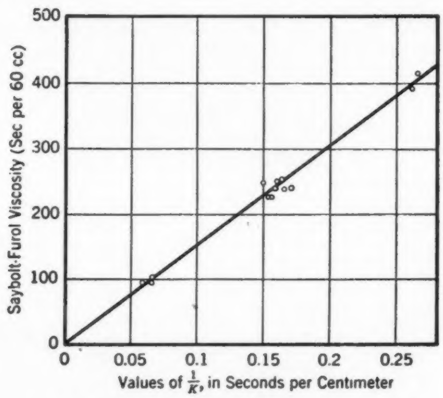


Fig. 8.—Relation Between Viscosity and Permeability

test on a sample taken from the discharge. The corresponding coefficient of permeability was then read from Fig. 8, which shows the relation between the viscosity and the reciprocal of the coefficient of permeability.

Although it would appear from Fig. 7 that the flow between the treated glass plates follows strictly the law of laminar flow with zero velocity along the walls, it is believed that the boundary velocity is not actually zero. This is concluded from the manner in which a film of the fluid will flow off the paraffin-covered surface, showing that there is no affinity between the fluid and the paraffin. The influence of this boundary velocity on the relationship between velocity and hydraulic gradient would merit an accurate investigation over a much larger range of hydraulic gradient.

To study the effect of base dimensions on drainage, models having a length-to-height ratio (L/H) varying from 1 to 40 were tested. Most of the models had a height of 6 in., and the largest model had a length of 20 ft. Longer models were impractical to construct in the laboratory. The models were tested with slopes varying between zero and 5%.

Photographs of a typical test, taken at two different elapsed times from start of drainage, are shown in Fig. 6. It was found convenient to present the test results by plotting values of $U\%$ versus the logarithm of time. Corresponding to the definition used in the theoretical analysis, $U\%$ is taken as the ratio (in percentage) of the volume of fluid drained after a given elapsed time to the total volume of fluid in the model at the beginning of the test.

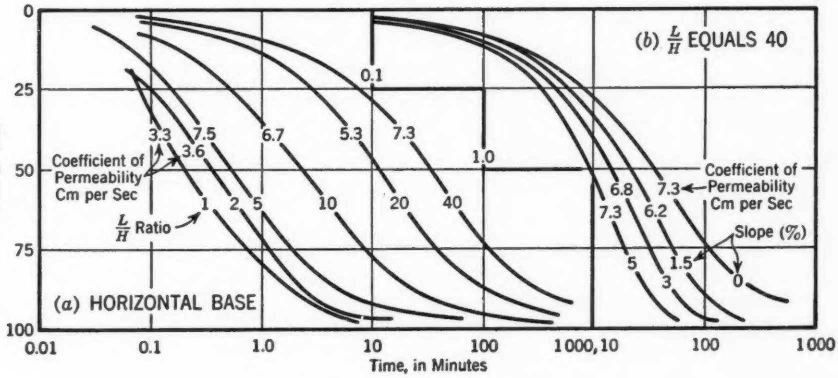


FIG. 9.—TYPICAL DRAINAGE CURVES

Fig. 9 shows typical test results and illustrates the effect of the ratio L/H and slope on drainage (%). Fig. 9(*a*) presents the results of six tests having ratios L/H ranging from 1 to 40, with horizontal base and approximately equal coefficients of permeability. Fig. 9(*b*) shows the results of four tests with slopes between zero and 5%, a ratio L/H of 40, and approximately equal coefficients of permeability.

FULL-SCALE TESTS

In addition to laboratory models, a full-scale field test installation was built in which base drainage under controlled conditions was studied. The test area consisted of four sections, numbered 1 to 4, each 10 ft wide, 75 ft long, and sloping 1.5%. The base materials and thicknesses used are listed in Table 1. Each test section was so constructed that its sides, top, bottom, and upper end were impervious. The impervious top was weighted with sand and gravel to resist the uplift forces during saturation of the base. At the lower end a drain was provided. Wells were provided to saturate the base and to observe the elevation of the water surface in the base during drainage tests.

The operation of a test section consisted of saturating the base with the drain closed, using water that was passed through a filter for de-airing, stopping the inflow of water, and simultaneously opening the drain. During drainage the quantity of water discharged, the elevation of the water surface at numerous locations in the base, and the temperature of the discharged water were measured. A typical profile for field test section 4, showing the water surface at various elapsed times, is shown in Fig. 10.

TABLE 1.—DATA FROM FIELD TESTS (SEE FIG. 15)

Test section No.	Base material	Slope (%)	Coefficient of Permeability, k, in Centimeters per Second					Effective Porosity n_e		
			Free Discharge Tests			Constant Velocity Tests[a]		Range		
			Average	Range From:	Range To:	Depth (in.)	k	Average	From:	To:
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
(a) SIX-INCH BASE: $H = 0.5$ FT; AND $L = 75$ FT										
1	Washed pea gravel	1.5	13	10	16	....	....	0.40	0.31	0.62
2	Sandy gravel	1.5	0.13	0.11	0.15	....	....	0.03	0.02	0.04
(b) EIGHTEEN-INCH BASE: $H = 1.5$ FT; AND $L = 75$ FT										
3	Washed pea gravel	1.46	16	13	21	....	....	0.43	0.41	0.46
4	Sandy gravel	1.63	0.27	0.21	0.33	15 9 4	0.33 0.074 0.027	0.07	0.05	0.09

[a] Coefficient k, in centimeters per second, for the different depths of flow, in inches.

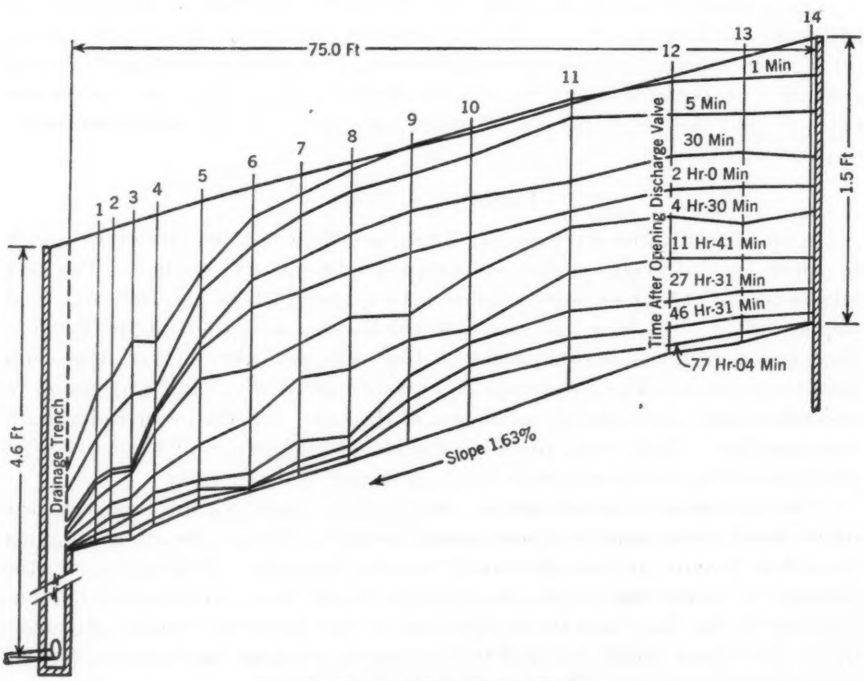


FIG. 10.—TYPICAL DRAW DOWN CURVES

For a comparison with the theory it was necessary to determine the coefficient of permeability of the base materials and their effective porosity. Determinations of coefficient of permeability were made by direct testing of the base. A trench was excavated the full width of the base at the upper end of each section, and de-aired water was introduced into the test section at this point until a steady state of flow was obtained. Two methods were used for performing the permeability test, these methods being the same in principle as the two methods used for the detemination of the coefficient of permeability for the laboratory models.

In the free discharge tests the coefficient of permeability was computed by means of two different procedures. One procedure was based on flow-net analysis, and the other method consisted of the use of a simplified formula in which an average hydraulic gradient and average cross section of flow was substituted in Darcy's law. Both procedures gave similar results, which are summarized in Table 1.

Constant velocity tests were performed on Section 4. The average values for the coefficient of permeability, as computed from these tests, are also summarized in Table 1. It should be noted that the coefficient of permeability shows a large decrease with a decreasing depth of flow. It is believed that this difference is due principally to downward segregation of fine particles during construction as well as during the drainage tests. In addition, the lower layers have received a greater degree of compaction.

The effective porosities of the base material in each test section, computed from the drainage tests, are listed in Table 1.

Comparison of Theory with Results of Experiments and Field Observations

Comparison with Laboratory Experiments.—In Fig. 11 are shown the semi-logarithmic plots of the theoretical curves for zero slope and for a slope factor of 0.5. For comparison, the observations from fluid model tests No. 1 and No. 4 with a ratio of $L/H = 20$ are also given. For these laboratory tests a plate spacing of 0.635 cm was used, the height (H) was 15 cm, the base width (L) was 609 cm, and the value of k was 7.3 cm per sec. The theoretical curves are matched through the 50% point. From this and numerous other comparisons between the theoretical and experimental results, it is concluded that Eqs. 6, 8, 11, and 13 match closely the shape of the experimental curves. The deviations are small and are probably of the same order of magnitude as the experimental error.

The position of the time curves on a semi-logarithmic plot is determined by the parameter c which was introduced in Eq. 2*b*. By comparing the time factor for 50% drainage with the actual time required for drainage, using Eq. 5, one can compute the parameter c that best represents given test results. For the two examples shown in Fig. 11, the parameters (c) of the observed curves are 1.5 and 2.4. A satisfactory average value for the conditions prevailing in airfield design is a value of c equal to 2, which happens to coincide with the value that had been recommended for design purposes before model tests were available.

In Fig. 12 the actual shape of the free surface corresponding to 50% drainage is compared with the assumed straight-line surface. As can be seen, the shape of the draw-down curve at any elapsed time differs radically from the straight

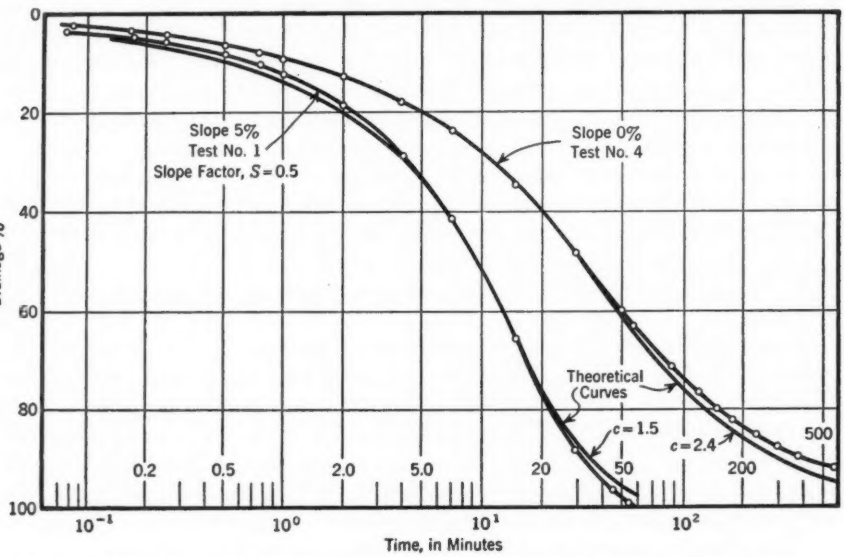


Fig. 11.—Comparison of Theoretical Drainage Curves with Laboratory Test Results

line assumed in the theoretical analysis. By means of such plots of flow nets for various stages of this nonsteady flow problem, it became apparent that the shape of the actual draw-down curve could best be approximated by ellipses.

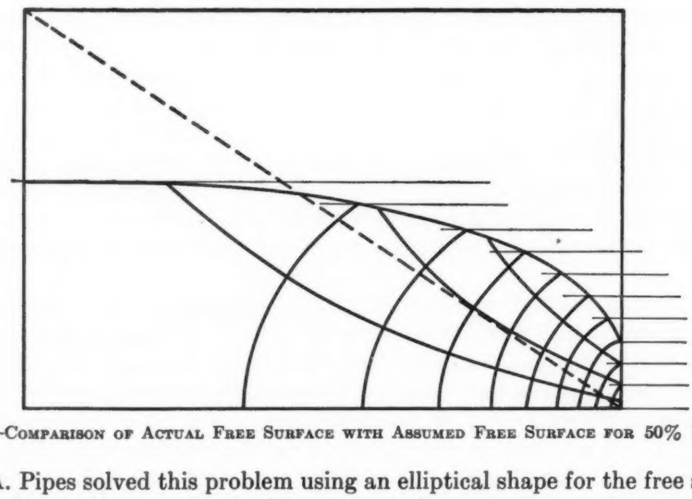


Fig. 12.—Comparison of Actual Free Surface with Assumed Free Surface for 50% Drainage

Louis A. Pipes solved this problem using an elliptical shape for the free surface. He found that the formulas resulting from such an assumption were essentially identical with the formulas derived on the basis of a straight-line assumption.

This paradoxical result can be understood by comparison with the simple Dupuit formula for the quantity of seepage through an earth dam. Although Dupuit's formula is based on a radical simplification of the free surface, quantitatively it checks very well with the few rigorous solutions that are available for

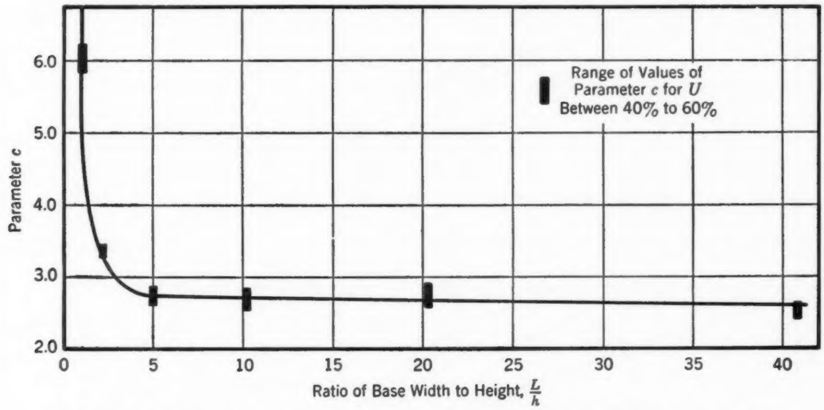


FIG. 13.—PARAMETER c COMPUTED FROM LABORATORY TESTS WITH HORIZONTAL BASE

the steady problem of seepage with a free surface. Generally, it may be said that whenever quantities of seepage are to be computed, even a rough approximation of the free surface will yield a good approximation of the quantity of seepage.

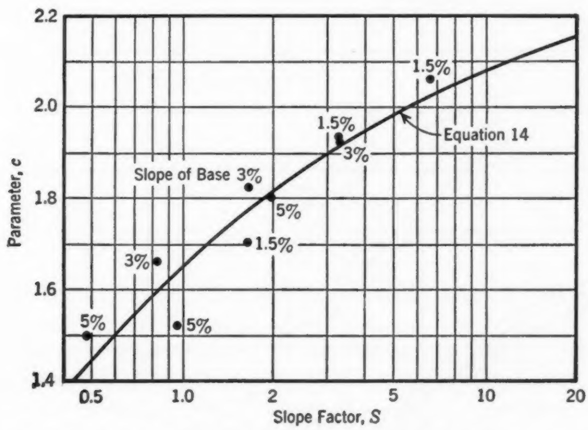


FIG. 14.—RELATION OF SLOPE FACTOR TO PARAMETER c DERIVED FROM VISCOUS MODEL TESTS, FOR L/H GREATER THAN 10

More detailed information on the relation between the ratio L/H and the parameter c is shown in Fig. 13 as derived from horizontal viscous-fluid model tests, and in Fig. 14 from tests on models with different slopes. In these figures the values of c were determined in each case as the arithmetic average of the

values of c for 40% drainage, 50% drainage, and 60% drainage. By using such an average, it is believed that a better approximation is obtained than by taking only one value that may be inordinately influenced by an individual error in reading.

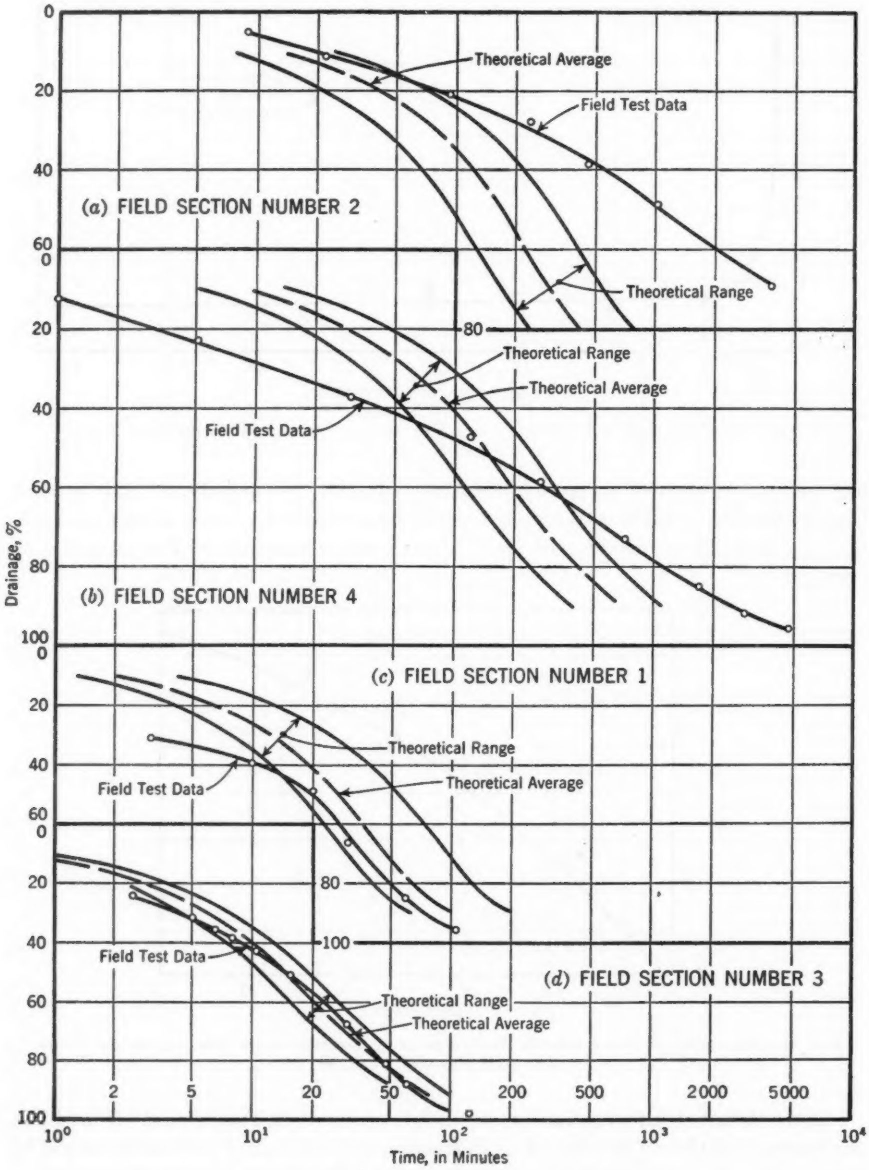


FIG. 15.—RESULTS OF FIELD DRAINAGE TESTS

In Fig. 13 it can be seen that for horizontal models and for ratios L/H greater than 5, the parameter c is essentially constant. For sloping models it is evident in Fig. 14 that the variable determining the parameter c is not the slope itself but the slope factor (S). Within the range of slope factors that are applicable in the design of airfields, the parameter c varies between 1.5 and 2.4, and it may be computed by an equation suggested by Helge Lundgren:

$$c = 2.45 - \frac{0.8}{\sqrt[3]{S}} \qquad (14)$$

Eq. 14 is the expression for the line plotted in Fig. 14.

Comparison with Full-Scale Tests.—In Fig. 15 the results of the drainage tests are compared with the theoretical drainage curves computed from Eqs. 11 and 13, and using the maximum and minimum values for the coefficient of perme-

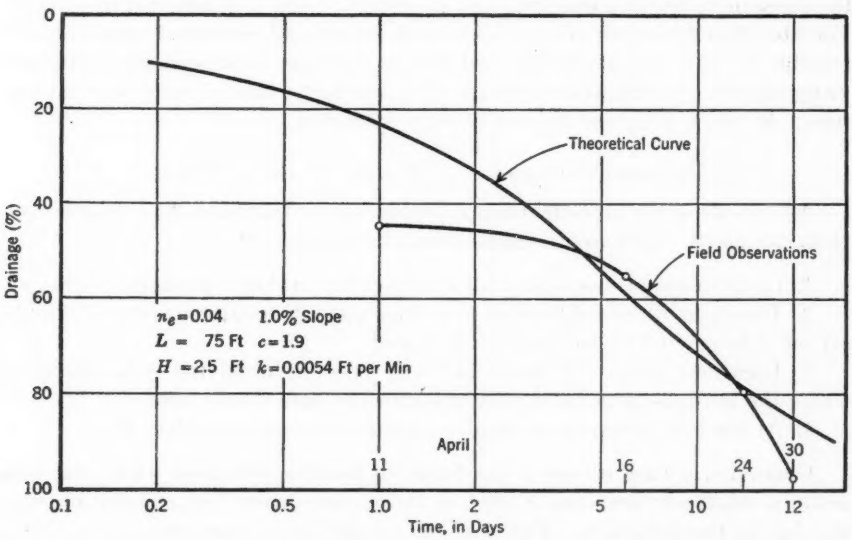


FIG. 16.—COMPARISON OF THEORETICAL DRAINAGE CURVE AND OBSERVED DRAINAGE CURVE AT SITE B

ability and effective porosity for the material as actually used in the tests. Table 1 lists pertinent data relating to the field tests and the curves of Fig. 15. For the tests with pea gravel (test sections No. 1 and 3 and Fig. 15(c) and 15(d)) the test data compare favorably with the theoretically computed range. In the case of section No. 2 (Fig. 15(a)) the test itself was considered unsatisfactory due to excessive leakage from the test flume and because the permeability of the base material, as placed, varied erratically. Test data on section No. 4 (Fig. 15(b)) do not compare satisfactorily with the computed theoretical range except at about 50% drainage. Since this test section represents a type of base material often used in airports, the reasons for this divergence would merit a detailed investigation. It is believed that the principal reasons for the divergence are the fact that the coefficient of permeability decreases with decreasing

average depth of flow (see Table 1) and the possibility that placement and compaction has resulted in stratification of the base. To some extent the appreciable thickness of the zone of capillary saturation also contributes to the divergence.

It should be noted that the divergence between theoretical and experimental results was found only in the well-graded sand and gravel that also contained a slight quantity of silt. The tests on washed pea gravel showed excellent agreement.

Comparison with Field Observations.—At site B, described in section 2, sufficient field data were obtained to construct a time-drainage curve and compare it, as shown in Fig. 16, with computed theoretical time-drainage curves using Eqs. 11 and 13. At site A, the base was fully saturated on April 16 and during the following forty days, 35% drainage of the base occurred. Using Eq. 11 the computed time for 35% drainage is about 50 days. For these two sites the agreement between observed and computed values is considered reasonable. For sites D and E, comparisons between observed and computed values are not possible because of complicated subsurface drainage and probable additional water supply by infiltration through the pavement while drainage was in progress. At site C the base did not become saturated.

Recommendations for Design of Base Drains

As a result of the investigations reported herein, the following recommendations are made, and base drainage should be required at:

1. Locations where considerable ice segregation in the subgrade is anticipated;
2. Locations where infiltration of surface water through a pavement may be expected to result in saturation of the base;
3. Locations where the pavement may become inundated occasionally or where the ground-water table may rise into the base course; and
4. At the low points of longitudinal grades sloping more than 2%.

Generally, a base course is not liable to become saturated where the subgrade is relatively pervious, except in those cases where ice segregation may develop in the subgrade. The requirement for base drains may be modified at locations where subsurface drains are provided to lower a ground-water table in the subgrade or to intercept water flowing in a pervious zone. Such drainage should be designed to provide also for drainage of the base course.

The principal factors that determine drainage design are the dimensions, the slope, the coefficient of permeability of the base, and the spacing of the drains. In the design, these factors may be varied to determine a practical solution. For example, it may be more economical to use a poorly draining base material combined with a close spacing of the drains than a more pervious but more expensive base material combined with much wider drain spacing.

For wide aprons and runways it may be necessary to provide several lines of base drains. However, for the usual runway and taxiway pavements, base drains at pavement edges only will usually prove adequate.

If base drains are to be installed, in the absence of special requirements it is recommended that these drains be so designed that the time required for 50%

drainage of the base course is not more than about 10 days. This recommendation is based primarily on the writers' investigations and experience. It is believed that base drainage designed in accordance with such a rule will be

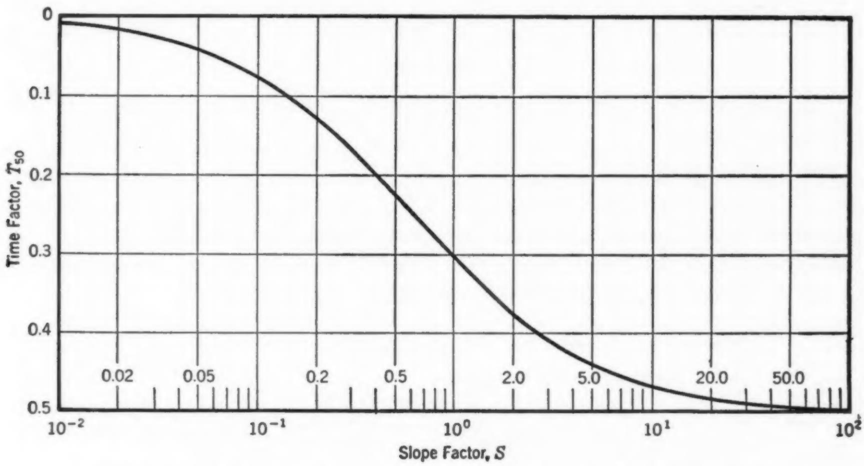


FIG. 17.—RELATION BETWEEN SLOPE FACTOR AND TIME FACTOR FOR 50% DRAINAGE

effective, yet not excessively costly. With this rule one of the following two procedures may be used:

Procedure I.—Fig. 17 is a plot of the relation between the time factor T_{50} and the slope factor S obtained from Eq. 11. This plot permits direct reading of the value of T_{50} for any given slope factor S. Using this value of T_{50}, the corresponding time (t_{50}) is then computed from

$$t_{50} = \frac{c\, n_e\, T_{50}\, L^2}{2\, k\, H} \qquad (15a)$$

in which the parameter c is taken from Fig. 14.

Procedure II.—For most practical purposes a further simplification is permissible by use of the following equation:

$$t_{50} = \frac{n_e\, L^2\, S}{2\, k\, H\, (S + 1)} = \frac{n_e\, L^2}{2\, k\, (H + L \tan \alpha)} \qquad (15b)$$

For the working range of H, L, and slope generally used in airfield design Eq. 15b was found to give within about $\pm$ 10% the same result as Procedure I. Eq. 15b has the advantage that it does not contain the empirical parameter c and that no charts are needed. The degree of approximation of Eq. 15b is illustrated in the following:

Slope factor, S	Ratio, R
0.2	0.94
1.0	1.00
10.0	1.14

in which, for the working range of slope factors S, the corresponding ratio—

$$R = \frac{t_{50} \text{ from Eq. 13}}{t_{50} \text{ from Eq. 20}} \qquad (16)$$

—is correlated.

Example

An application of the recommended design procedure is illustrated in the following example:

Runways 300 ft wide, for heavy airplanes, are to be built on a soft clay subgrade, as shown by the cross section in Fig. 18. Because of local conditions occasional saturation of the base is considered possible. The base course, 4.5 ft thick, consists of sand and gravel with an average coefficient of permeability of $k = 0.01$ ft per min and an effective porosity, $n_e = 0.1$.

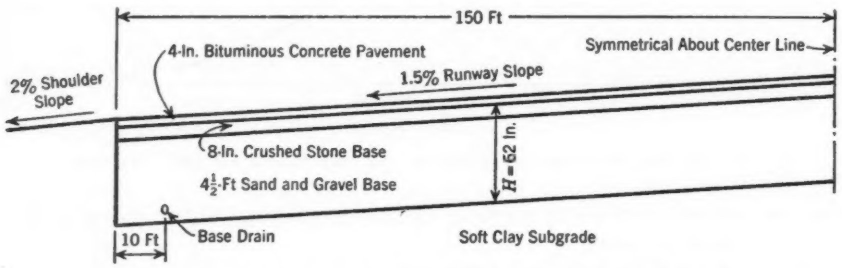


Fig. 18.—Example of Design of Base Drains Along Edge of Runway

The rule of 50% drainage in 10 days is assumed for the design of the base drainage. Because of the thickness of the base, the drains may be placed in the base, near the bottom, as shown in Fig. 18, thus making $L = 140$ ft. For the drainage analysis the depth of the base is assumed to be the thickness of the sand and gravel and crushed stone ($H = 62$ in.). The coefficient of permeability for the entire layer is assumed equal to that of the sand and gravel, an assumption which is on the conservative side.

To determine the suitability of the section, compute from Eq. 10, $S = 2.46$; from Fig. 17, $T_{50} = 0.395$; from Fig. 14, $c = 1.85$; and from Eq. 15a, $t_{50} = 9.6$ days. Hence, two lines of base drains, each 10 ft inside the pavement edge as shown in Fig. 18, are considered satisfactory.

During construction and before the pavement is placed, the base drains will drain water infiltrating into the base due to precipitation. Complete saturation of the base will probably result during each period in which the precipitation exceeds the storage in the base. Therefore, the question is pertinent as to how quickly construction operation can be resumed after a rain storm. It is considered reasonable to assume that construction can proceed when the base is only 10% drained. For instance, if the base is partly constructed to a depth of 2 ft, 10% drainage would require about 29 hr, as computed using Eqs. 11 and 5. Hence, the proposed drains are believed to provide satisfactory drainage of the base during construction.

ACKNOWLEDGMENT

The investigations described in this paper were started in 1945 by the New England Division Office, Corps of Engineers, for the Airfields Branch of the Office, Chief of Engineers, in connection with a comprehensive study of criteria for the design of base drainage and subsurface drainage of airport pavements. They are described in detail in several unpublished reports by the New England Division.

As a member of the Board of Consultants on Airfield Pavements for the Office, Chief of Engineers, the senior writer made the theoretical studies and suggested the use of the viscous fluid models. The junior writer, as chief of the Frost Effects Laboratory, New England Division Office, was in charge of the laboratory and field investigations during the period 1945 to 1947.